APPENDIX A
FORT PECK SPILLWAY
MAJOR REHABILITATION STUDY

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Ft. Peck Spillway

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Major Rehabilitation Study

August 2000

US Army Corps of Engineers
Omaha District

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FT. PECK DAM – FT. PECK LAKE
MISSOURI RIVER, MONTANA

SPILLWAY MAJOR REHABILITATION STUDY

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Fort Peck Spillway Preliminary Major Rehabilitation Study

1. Fort Peck Project Background. Flow releases from the Fort Peck project are possible through the power plant, outlet works, and spillway. A preliminary investigation was conducted to determine the reliability of the spillway during operation and to assess potential damage as a result of operation. The Fort Peck spillway contains no provision for energy dissipation at the spillway exit channel. The downstream spillway chute has experienced pronounced movement since monitoring of the structure began in 1940. Using information provided within the reconnaissance study, estimates of failure mechanisms and the flow rate at which failure occurs were performed.

An estimation of damage which may result during spillway flows at Fort Peck was performed. The *Fort Peck Dam Spillway, Engineering Reconnaissance Study, August 1996*, addressed operating deficiencies with the spillway. Damage estimates for various flow rates are based on the findings within the reconnaissance study prepared by R.W. Beck.

2. Release Mechanisms. Releases from the Fort Peck project are possible through the power tunnels, outlet works, spillway and spillway gate overtopping during unregulated release.

2.1. Power Tunnels. Normal releases are through the 2 power tunnels (5 generating units) with a capacity of approximately 15,000 cfs at the rated head. At the maximum operating pool elevation of 2250, the power tunnels total discharge capacity is 17,820 cfs.

2.2. Flood Tunnels. The 2 flood tunnels are each regulated with a ring gate with a capacity of approximately 22,500 cfs per tunnel at a pool elevation of 2250. The *Major Rehabilitation Evaluation Report, Outlet Works Modifications, March 1994*, addressed operating deficiencies with the outlet works.

2.3. Spillway. The spillway is regulated with 16 stoney vertical lift gates each 40 feet wide by 25 feet in height. For a pool elevation of 2250.2, the discharge through a single gate with a 1-foot opening is 1040 cfs or 16,640 through all 16 gates. The spillway crest elevation is 2225 feet.

2.4. Spillway Gate Overtopping Flow. Due to operational constraints, overtopping of the Fort Peck spillway gates may occur. Flow over the top of the gates may be roughly approximated as weir flow. With 16 gates, each 40 foot wide (total length of 640 feet) and a weir coefficient of 3.0, the total overtopping flow rate is 1900 cfs for 1 foot of head, 5400 cfs for 2 foot of head, and 10,000 cfs for 3 foot of head.

3. Operating Concerns. Operating concerns during high pools consist of overtopping the spillway gates and the procedure to safely release flows in excess of power capacity from the project.

3.1. Gate Overtopping. If gate overtopping above a certain height must be prevented, raising all the gates 1 foot releases a much higher flow (16,640 cfs compared to only approximately 2000 cfs if all the gates overtop by 1 foot). Therefore, increasing releases prior to overtopping of the gates is preferred to prevent having to raise all the gates simultaneously.

3.2. Outlet Works Release. Past operating experience has indicated that gate damage has occurred as a result of releasing flow. Gate vibration and fatigue are concerns. Previous operation has required periodic maintenance and inspection. The ring gate in tunnel 3 was improved in the 1960's. However, tests conducted by WES and operation in 1975 indicate that the ring gate is still damaged during flow. No data is available which conclusively indicates that a preferred flow range is available which results in the least damaging condition. Studies do indicate that cavitation is the least in the fully vented condition.

3.3. Spillway Releases. The spillway engineering recon study identified concerns with the spillway. Damage due to cavitation, uplift on spillway slabs, and scour at the spillway exit were evaluated.

4. Description of Spillway Features. The Fort Peck spillway is located remotely from the project, approximately 3 miles east of the main embankment, in the right abutment. The spillway consists of a partially lined approach channel, a gated control structure, and a partially lined discharge channel which enters the Missouri River approximately 9 miles below the dam. The spillway was constructed within an outcrop of Bearpaw shale. Near the surface, the shale has weathered extensively. Numerous faults have been identified in the spillway area. Immediately downstream of the lined spillway exit channel, the un-lined channel has been enlarged and deepened by erosion. Provisions for energy dissipation downstream of the concrete lined channel were not included within the constructed project.

4.1. Spillway Location. Four possible spillway sites on the right bluff of the Missouri River were originally studied. The selection of the present site was based on the distance of the gate structure from the dam, the location of the outlet channel with respect to the downstream toe of the dam, and the long approach channel giving an additional factor of safety in the event of failure of the gate structure.

4.2. Control Structure and Approach Channel. The approach channel is nearly 2000 feet in length and is composed of a lined channel for 360 feet upstream of the control gates and 1600 feet of unlined channel. Flow within the spillway channel is regulated by 16 vertical lift gates which are each 25 feet in height by 40 feet in width. The gates are electrically operated and can be individually operated. A 10-foot wide by 30-foot deep concrete cutoff wall lies beneath the upstream edge of the control gate foundation.

4.3. Downstream Spillway Chute. The downstream spillway channel includes a concrete lined channel for a length of 5030 feet. The floor slab of the concrete lined discharge channel varies from 2.33 to 4.0 feet thick. Floor slab sections are 20 feet wide in the longitudinal channel direction and generally 30-40 feet wide in the transverse direction. Downstream of the gate structure, the channel converges from a width of 800 feet at the gate structure to a bottom width of 120 feet at the spillway exit. The overall vertical drop from the crest at the gate structure to the spillway channel exit is 214 feet. For the lower approximately 4000 feet of the lined spillway exit channel, the bottom slope is a constant .0523 ft/ft.

4.4. Cutoff Wall. The lined channel terminates at elevation 2011.0 feet msl with a cutoff wall. The cutoff wall structure is cellular, extends to a depth of 70 feet below the spillway channel invert to elevation 1941.0 feet msl, and also includes wingwalls. The main section of the cutoff structure which spans the channel is 229 feet wide. The wingwalls extend 260 feet at an angle of 45 degrees (185 feet in the direction perpendicular to flow). Total cutoff wall span, measured perpendicular to the direction of flow, is 600 feet. Cellular cutoff wall length, measured in the direction of flow, is 95 feet.

4.5. Downstream Unlined Channel. Downstream of the spillway channel chute and cutoff wall, an unlined discharge channel continues for a length of approximately 2700 feet to the Missouri River. Original construction included excavation through the shale bluffs to the Missouri River floodplain. Channel excavation consisted of a bottom width of 130 feet, side slopes of 2H on 1V, and a flat gradient at an elevation of 2010. After exiting the shale bluff, a 12-foot wide pilot channel was excavated through the river floodplain to the Missouri River. Following construction, spillway flows have altered the channel section and grade within the unlined exit channel.

4.6. Energy Dissipation Structures. Preliminary design of energy dissipation structures which could be employed at the Fort Peck spillway channel exit were performed by the Omaha District Hydraulic Section in the 1960's. Energy dissipation structures considered included a conventional stilling basin and a flip bucket. Available data in Hydraulic Section files indicates that design of an energy dissipation structure was not finalized. Due to the limited detail provided within the design, a cost estimate was not performed. However, either a flip bucket or conventional stilling basin should be regarded as an effective alternative of dissipating energy downstream of the spillway exit and limiting scour depth to an acceptable level.

5. Geology and Foundation. A through discussion of spillway area geology including boring logs and geologic sections are provided in Design Memorandum MFP-118 (Omaha COE, 1973). General information from the report is summarized in this section. Assessment of the rock strength is a necessary parameter in determining scour depth below the spillway. Bedrock in the Fort Peck area is the Bearpaw shale. This is a compaction type shale consisting of dark gray to black clay shale made up of marine sediments. It is comparatively thin bedded and beds of bentonite occur at different intervals. Weathering disintegrates the shale considerably.

As discussed in Design Memorandum MFP-118, 1973, a number of holes were drilled through the slab for the primary propose of determining the condition of the shale immediately under the slab. Extreme fracturing was detected in the first two feet below the slab. Some fracturing accompanied by extensive jointing occur in the shale to a depth of 8 to 10 feet and fairly abundant jointing but no fracturing to a depth of approximately 30 feet.

6. Spillway Flow Parameters. Regulation of the probable maximum flood results in a peak spillway discharge in excess of 250,000 cfs for a duration of approximately 4 days. The spillway flow computations performed within the spillway recon study were used to estimate flow depth and velocity. Due to the changing slope and converging bottom width, the flow velocity and depth vary for different spillway locations at a constant discharge.

6.1. Computed Flow Parameters. The lower portion of the spillway channel has a fairly constant geometry. Within the recon study, computations were performed employing a rugosity or roughness height of .002 and .007 feet. During the 1946 spillway observations, the velocity between stations 40+00 and 45+00 was determined to be about 62.5 feet per second (fps) at a discharge of 27,000 cfs. The measured velocity corresponds fairly well with computed velocities. A summary of computed flow parameters is shown in Table 1. The computed flow parameters illustrates that the spillway flow velocity for the lower 3000 feet of spillway exceeds 40 ft/sec for all flows which were computed (25,000 - 265,000 cfs).

Table 1.
Spillway Flow Computations
Minimum Roughness Height = .002 feet

	Station 20+00		**Station 52+20 Spillway Exit**	
Flow (cfs)	Depth (ft)	Velocity (ft/sec)	Depth (ft)	Velocity (ft/sec)
25,000	3.1	38	3.7	55
75,000	7.2	45	4.6	65
125,000	11.5	48	11.5	83
175,000	15.2	49	14.2	89
265,000	22	50	20	94

6.2. Release Duration Relationship. When flood control releases are required in excess of the power plant capacity (approximately 15,000 cfs) due to reservoir operating criteria, releases may be made through either the spillway or the outlet works. Table 2 lists the number of days the spillway flow would equal or exceed the given value for conditions without releases through the outlet works.

Table 2. Number of Days Spillway Flow Exceeds Given Value		
Spillway Flow (cfs)	**Standard Project Flood (days)**	**Spillway Design Flood (days)**
20,000	22	33
50,000	17	30
100,000	6	21
200,000	0	10

The data listed in Table 2 provide a general assessment of the duration of spillway discharges. For the purposes of this study, the outlet works were considered inoperable due to operational constraints imposed by the Missouri River Region office.

7. Spillway Damage Mechanisms. The spillway engineering recon study identified several different mechanisms by which spillway damage may occur. Damage due to cavitation, uplift of the spillway slabs, and scour at the spillway exit were evaluated. The flow rate at which spillway damage begins and the type of damage which occurs varies with the different damage methods.

7.1. Scour. Below a spillway flow of 60,000 cfs, the cut-off wall and wingwalls are expected to be safe. In the range of 60,000 cfs to 125,000 cfs, the project may function as designed. Above 125,000 cfs, the wingwalls at the spillway exit are expected to fail and the lower end of the spillway will be damaged.

7.2. Cavitation. Major cavitation damage is not expected to occur for the offsets which were measured in the existing condition.

7.3. Slab Uplift. Due to the prolonged period expected for spillway flows and the movement which the spillway has experienced since construction, the spillway is not considered to be watertight. Slab uplift pressure may be expressed as a percentage of the velocity head based on the vertical offset and joint width opening. The downward force resisting uplift consists of the slab weight and weight of the water. The spillway drains relieve the uplift pressure. If the spillway drains are inoperable, the slab has a safety factor less than 1 for the lower 2000 feet of spillway at a flow rate of 25,000 cfs. For a safety factor of 1, the estimated required spillway drain efficiency is between 50 and 80%.

8. Spillway Damage Estimate. Estimating damage to the cutoff wall is extremely difficult given the unknowns of the cutoff wall strength. Forces acting on the wall during flow are nearly impossible to determine. Preliminary damage estimates to the cutoff wall and spillway chute channel were made based on an erosion depth. A detailed analysis of the durability of the wall with respect to scour hole formation was beyond the scope of this study. Plans of the cutoff wall illustrate that it is a massive structure. Damage was determined assuming the wall should be fairly resistant to breakup as the protecting rock layer is scoured away from the front wall face and from battering by loose rock during the scouring process. Based on engineering judgement, damage was broken into four categories of none, minor, major, and replacement. Costs were computed for each damage level based on the damage category. Factors which affect the damage include the high spillway flow velocity for all flows, the observed upheaval areas in the spillway chute, and the poor quality material beneath the spillway slab. These factors all indicate that a single slab failure could quickly propagate upstream and affect a significant portion of the spillway.

8.1. No Damage. For minimal erosion depths, the cutoff wall may suffer some superficial, exterior damage that will not require repair.

8.2 Minor Damage. Minor damage to the cutoff wall and lower end of the spillway chute may occur during lower flow events. Damage costs for this range of scour depth was based on assuming that, although the majority of the cutoff wall remains intact and functional, portions of the wingwall or spillway chute may require reinforcement or replacement. The minor damage scenario would consist of an assumed 20% damage to spillway slabs (10' x 20') which translates into 501 slabs damaged. The damage is assumed to be random. The concrete slabs would be removed from the site and the new slabs constructed and anchors installed.

8.3 Major Damage. Major damage to the cutoff wall and lower end of the spillway chute may occur during medium flow events. Damage was based on the assumption that the cutoff wall would retain enough strength to prevent complete destruction of the spillway channel. However, damage to the cutoff and wingwall structure would be severe. The major damage scenario would consist of loss of all slabs from Station 20+00 to end of spillway, and loss of wingwalls. The failed structural concrete would be removed from the site, new slabs installed, anchors installed, a slab drain system installed, and the wingwalls reconstructed.

8.4 Total Failure. Failure of the wingwall and cutoff wall structure was determined to be probable for large flow events. In this case, the wall would no longer serve to protect the spillway chute channel from erosive forces and undermining. Previous studies have determined that the shale immediately under the spillway slabs is highly deteriorated. Much like a headcut, the erosion <u>may progress upstream fairly rapidly and fail the entire spillway chute channel.</u>

The total failure *scenario #1* would consist of loss of entire spillway from Station 20+00 to the end of the spillway (slabs, wingwalls, cutoff walls). The failed structural concrete would be removed from the site, scour hole soil regraded, new slabs installed, wingwalls and cutoff wall reconstructed, drain system added, slab anchors installed.

The total failure *scenario #2* would consist of loss of entire spillway from Station 20+00 to the end of the spillway (slabs, wingwalls, cutoff walls). The repair would consist of failed structural concrete removed from the spillway, new slabs installed, anchors installed, a slab drain system installed, and stilling basin constructed.

The total failure *scenario #3* would consist of loss of entire spillway from Station 20+00 to the end of the spillway. The repair would consist of failed structural concrete removed from the site, existing slab removal for flip bucket construction, and flip bucket and wingwalls and cutoff wall constructed.

8.5 Rehabilitation - Concept #1. The rehabilitation concept #1 of the spillway will consist of a fix to the existing spillway to alleviate problems as detailed in the scenarios stated above. The rehab scope would consist of slab anchors installed and stilling basin constructed.

8.6 Rehabilitation - Concept #2. The rehabilitation concept #2 of the spillway will consist of a fix to the existing spillway to alleviate problems as detailed in the scenarios stated above. The rehab scope would consist of slab anchors installed and flip bucket constructed.

8.7 Rehabilitation - Concept #3. The rehabilitation concept #3 of the spillway will consist of a fix to the existing spillway to alleviate problems as detailed in the scenarios stated above. The rehab scope would consist of joint sealing system installed and flip bucket constructed.

8.8 Quantities. The following are the major quantities that were calculated for the various damage and rehabilitation schemes.

Minor:

Replace slabs- 501,000 SF x 20% = 100,200 SF
(10' x 20' slabs, 200 SF)
100,200 SF/200 SF = 501 slabs - <u>8,660 CY</u> of Reinforced Concrete
<u>6012</u> new anchors, (12 per slab)

Major:

501,000 SF of slabs 2'-4" thick- 10' x20'
501,000 SF x 2.33' = 1,168,833 CF = <u>43,290 CY</u> of concrete
Anchors- 501,000/16 SF spacing = <u>31,312</u> anchors
Drains- <u>9600 LF of 18"</u> mains, <u>27,000 LF of 8"</u> laterals
gravel- 3' x 3' x 27,000' = 243,000 CF = 9,000 CY = <u>15,300 Tons</u>
(4'+14'/2)x5"= 432,000 CF = 16,000 CY = <u>27,200 Tons</u>
Wingwalls- <u>56,450 CY</u> of concrete
<u>30,000 CY</u> of excavation & backfill
Cleanup- <u>99,740 CY</u> of concrete from above
<u>64,000 CY</u> of scour hole soil

Total Failure:

Scenario #1:

Slabs- 43,290 CY of concrete

anchors- 31,312

drain system- 9,600 LF of 18" main, 27,000 LF of 8" laterals, gravel- 42,500 Tons

Wingwalls- 56,450 CY of concrete

Cutoff walls- 28,000 CY of concrete

Excavation & Backfill- 30,000 CY of excavation & backfill

Cleanup- 127,740 CY of concrete

127,134 CY of scour hole soil

Scenario #2:

Slabs- same as scenario #1

Stilling Basin- provided previously

Excavation- 600,000 CY, Backfill- 50,000 CY, Waste- 50,000 CY

Dewatering

Riprap- 12,850 Tons

Bedding- 5,150 Tons

Cleanup- same as scenario #1

Scenario #3:

Slab Removal- 100' x 220' x 2.33' = 51,260 CF = 1,898 CY

Flip Bucket- 4,678 CY of reinforced concrete

Cutoff Wall & Wingwalls- 42,225 CY of reinforced concrete

7,000 CY of excavation and backfill

Cleanup- same as scenario #1 & #2

Rehabilitation:

Concept #1

Anchors- 31,312

Stilling Basin- same as TOTAL FAILURE Scenario #2 excluding cleanup costs.

Concept #2

Anchors- 31,312

Flip Bucket- same as TOTAL FAILURE Scenario #3 excluding cleanup costs.

Includes 2/3 of Cutoff Wall & Wingwall Damage and ½ of Excavation & Backfill of TOTAL FAILURE Scenario #3

Concept #3

No anchors

Seal Joints

Flip Bucket- same as TOTAL FAILURE Scenario #3 excluding cleanup costs.

Includes 2/3 of Cutoff Wall & Wingwall Damage and ½ of Excavation & Backfill of TOTAL FAILURE Scenario #3

9. Description of Major Components. The major components included in the above rehabilitation scenarios are stilling basin, flip bucket, slab anchors and joint sealing system. These components are described in detail below.

9.1 Stilling Basin. The conceptual concrete stilling basin would start at the end of the existing spillway and terminate 467 feet downstream. The basin would be a uniform 110 feet wide. The basin invert would begin at existing elevation 2011 and drop on an elliptical slope to invert elevation 1950, it then continue for 250 feet at that same elevation. The last 17 feet would consist of an end sill at elevation 1957. The wingwalls would slope down from the existing 2070 to 2050 at 200 feet downstream, then continue at elevation 2050 for the last 267 feet. The wingwalls would be A-shaped have a maximum height of 100 feet. The construction work would involve dewatering the existing scour hole and excavating to desired grade and backfilling. The channel downstream would be provided with stone protection.

9.2 Flip Bucket. The conceptual concrete flip bucket would be located at the end of the existing spillway and have a vertical radius of 210 feet. The bucket would have a base length of 76.2 feet, be 14.7 feet high the end of the flip, and cover the entire 110 feet spillway width. The exit angle of the flip would be 18.4 degrees. The existing concrete slab would be removed and the flip bucket doweled to the concrete.

9.3 Slab Anchors. The anchors consist of a 1 3/8 inch diameter steel rods 17 feet long. The existing slab would be cut and jack-hammered, then bored and the boring continued into the shale foundation. The rod installed and grouted, a plate and tightener added at the slab end and the area covered with concrete. The anchor bar would embedded in at least 10 feet of competent shale. Each anchor would cover an area of 16 square feet.

9.4 Joint Sealing System. The spillway slab is not expected to be watertight since aging of the spillway has probably reduced the effectiveness of the water stops. A joint sealing system able to withstand 10 psi at a spillway flow of 125,000 cfs is required to prevent uplifting of the slabs. The following system is the best choice but it will have to be field or laboratory tested to verify. The manufacturer indicated it has been used in storage tanks with this and greater head (static condition). The system is described in the following paragraphs.

9.4.1 Description. EMSEAL Joint Systems, Ltd. is the manufacturer of 20H SYSTEM. 20H is a preformed expanding foam sealant produced by impregnating permanently elastic, high-density, open-cell polyurethane foam with water-based, polymer-modified asphalt. Partially filling the open-cells with the impregnation and then compressing the material results in levels of sealing depending on the degree of compression. Typically, approximately 5-times compression is required for water-tightness in below-grade and horizontal deck applications. The 20H foam is packaged precompressed in shrink-wrapped lengths. It is supplied precompressed to less than the nominal material size for easy insertion into the joint. Sealing between the foam and substrate is achieved through a combination of the effects of foam backpressure and epoxy adhesive applied to the substrates and into which the 20H foam is installed. The exposed outer surface of the installed 20H is further treated with TOPCOAT, supplied to suit the application. The complete 20H SYSTEM comprised three elements: 1) the 20H foam, 2) the epoxy adhesive, and 3) the TOPCOAT.

9.4.2 Joint Seal Characteristics. Below-grade and horizontal deck applications generally require compression to approximately 20% of the material's original uncompressed dimension (i.e. 5-times compression). The 20H is rated for joint movement of +25%, -25% (total 50%) of nominal material width. The following Table 3 gives the physical properties of 20H:

Table 3
Joint Seal Physical Properties

Property	Value	Test Method
Density (uncompressed)	9-10 lb/ft.^{3}	
Density (compressed to 20% of uncompressed width)	45-50 lb/ft.^{3}	
Tensile Strength	21 psi min.	ASTM D3574
Elongation – ultimate	150% min.	ASTM D3574
Temperature range		ASTM C711
High – permanent	185°F	
High – short term	203°F	
Low	-40°F	
Softening Point	140°F min.	ASTM D816
UV resistance	Excellent	
Resistance to aging	Excellent	
Low temperature flexibility 32°F to -10°F	No cracking or splitting	ASTM C711

10. Cost Estimate. MCACES estimates of minor, major and total failure scenarios, and rehabilitation concepts. Tabulated cost values are highly subjective and are based on engineering judgement regarding the extent of damage. The MCACES estimate is enclosed as Appendix B; the rehabilitation total cost is $75,642,000 for concept #1, $55,054,000 for concept #2 and $17,572,000 for concept #3. The total damage costs are shown in Table 4.

11. Monte Carlo Model Input. The final product of the spillway damage analysis was provided as input to the Monte Carlo model analysis. Input to the Monte Carlo model was provided in the format of damage for a given spillway flow rate. Four separate spillway flow ranges and associated costs were specified. Damage and flow was determined based on the results of the computed scour depth and damage estimates described in the spillway recon study. Spillway flow is determined within the Monte Carlo model based on pool level and the reservoir operating rule curve. Input provided to the Monte Carlo model is shown in Table 4.

Table 4 Fort Peck Spillway Monte Carlo Model Input Flow vs. Cost			
Spillway Peak Flow Range (cfs)	**Damage Category**	**Damage Probability**	**Damage Cost ($)**
0 - 60,000	None Minor Major Total Failure	1.0 0 0 0	3,000 10,159,000 84,302,000 101,073,000*
60,000 - 80,000	None Minor Major Total Failure	0.5 0.5 0 0	3,000 10,159,000 84,302,000 101,073,000*
80,000 - 100,000	None Minor Major Total Failure	0.1 0.5 0.4 0	3,000 10,159,000 84,302,000 101,073,000*
100,000 - 125,000	None Minor Major Total Failure	0 0.4 0.5 0.1	3,000 10,159,000 84,302,000 101,073,000*
> 125,000	None Minor Major Total Failure	0 0 0.1 0.9	3,000 10,159,000 84,302,000 101,073,000*

*The same Damage Cost was used for Total Failure Scenario's #1, #2 and #3.

12. Risk Based Analysis

12.1 Introduction. As part of the risk based analysis of the overall Fort Peck Project, the frequency and consequences of spillway uses were required to determine the economic effectiveness of any spillway rehabilitation measures. In order to estimate and combine the frequency of spillway uses with the damages to the spillway, a Monte Carlo simulation of the operation of the reservoir was required.

12.2 Monte Carlo Simulation. Monte Carlo simulation, in general, is a method to determine the probability distribution of the output of a system given the probability distribution of the inputs to the system. The three steps usually required in a Monte Carlo simulation are: determination of the input probability distribution, transforming the input into an output distribution, and analyzing the output. The Monte Carlo simulation for Fort Peck is required to determine the probability distribution of reservoir damage costs given the probability distribution of driving variables (reservoir pool levels, reservoir inflows, reservoir operating uncertainties, etc.). The simulation model developed for Fort Peck allows for both stochastic and deterministic elements of the reservoir system to be modeled. The stochastic elements involve the random variables of the system such as reservoir pool levels and reservoir inflows. The deterministic components of the system are operating uncertainties such as determining required release rates and methods of releases. A summary of the Fort Peck Monte Carlo Simulation Model is given below.

a. The model selects the annual maximum pool level by using a simple autoregressive model based on historical maximum annual pool levels.

b. The model selects the annual maximum daily inflow by selecting a uniform random number between 0 and 1 to represent the inflow probability of occurrence and applies that random number to the inflow-probability cumulative distribution function.

c. For the specified pool and inflow, the required release rate is determined by the model from existing reservoir operating rule curves.

d. For the given pool stage and release rate, the model utilizes an event tree to determine which release mechanism (power plant, outlet works, and spillway) will be used, the consequences of use, and the damage costs associated with the releases.

e. The model repeats for a specified number of years of simulation. The specified period of simulation is broken down into 50-year periods. Within each 50-year period, the damage costs are converted to present value. The average present value damage cost is calculated by averaging the present value damage costs for all 50-year periods.

f. The model is run for with and without project conditions. The reduction in average present value damage costs for with and without project conditions is divided by the project cost to determine the benefit-cost ratio.

12.3 Reservoir Pool Levels. The first step in the Monte-Carlo simulation was generating the time series of annual maximum pool levels for Fort Peck. Due to carry-over storage in the reservoir, the pool levels in Fort Peck are not independent. Because of the annual dependency of pool levels, a simple first order auto regressive model was used to simulate the time series of pool levels. The first order autoregressive model uses the following relationships (Salas, 1980):

$$X_t = X_m + Z_t$$

where:

X_t = generated pool level

X_m = historic mean pool level

$Z_t = r_1 Z_{t-1} + \Phi_> >_t$

Definitions:

A. r_1 - lag-1 serial correlation coefficient. Is a measure of the degree of linear dependence among successive values of a series (Salas, 1993).

$r_1 = C_1/Co$

where:

$c_k = (l/N)3(x_{t+k}-x_m)\,(x_t-x_m)$ *where* k=O,1 *N=number of samples*

The serial correlation coefficient is biased and can be corrected by using:

$r_1 = (\ 1 + Nr_1\ ) / (N-4\ )$

B. Variance of normal random variable

$\Phi^2_>$ - variance of normal random variable

$\Phi^2_> = \Phi^2\ (\ 1 - r_1^2\ )$

where Φ^2 = variance of the annual maximum pool levels.

C. $>_t$ - random standardized normal variable

Allows for random or noise element to the generated data.

Twenty-five years of data (1968-1992) since the Mainstem Missouri River Reservoir system has operated as a complete system were used in the development of the autoregressive model. The twenty-five years of data result in a mean pool level of 2240 ft msl, a biased lag1 serial correlation coefficient of 0.78, and an unbiased lag-1 serial correlation coefficient of 0.98. The model works by first selecting a uniform random number between O and 1. This number represents the cumulative probability of the standard normal distribution. A polynomial was used to approximate the standard normal variable (t,) associated with the randomly selected cumulative probability. The initial z, was set at zero so that the model will start at the historic mean maximum annual pool level. The model can then begin generating a series of pool levels that are dependent on the previous year's pool level. A correction factor was applied to the generated pool levels to insure that the generated pool level probability curve would match the historic pool level probability curve.

12.4 Reservoir Inflows. The model generates the annual maximum daily inflow into Fort Peck using the probability integral transform (Benjamin and Cornell, 1970). This involves selecting a uniform random number between O and 1 to represent the cumulative probability distribution quartile value. The randomly selected number is applied to the inflow-probability curve to generate the maximum daily inflow. The model also develops a generated inflow-probability relationship based upon Weibull plotting positions for comparison with the historic inflow-probability curve to insure that the relationship is not biased.

12.5 Reservoir Releases. The required releases for the generated pool levels and inflows were modeled according to the Emergency Regulation Curves - Late Spring Flood Season. The model rounds the generated pool level to the nearest foot and then interpolates between the inflow values to determine the release rate.

12.6 Event Tree. For the given required release rate for each year, the model selects the release system (power plant, outlet works, and spillway), consequences of the use, and damage costs associated with the release. The Monte Carlo simulation model essentially runs through the event tree for the worst flood event for each year and totals up the damage costs for that flood event. The event tree first determines whether there is a normal operating condition or whether there is an emergency condition which requires a rapid drawdown of the pool. For emergency conditions, all release mechanisms are modeled as being fully opened. The spillway flows for the emergency conditions, consequently, are the spillway capacity for the given pool elevation. For normal operating conditions, the method of release is prioritized in that for the given release rate, the model will first use the power plants if they are available. If the power plants are not available, the model will go directly to the spillway or outlet works for releases, depending on pool elevations and user specified release preference. If the required release rate exceeds the power plant release capacity for the particular pool level, the residual required release rate above the powerplants' capacity is sent to either the spillway or outlet works depending on the pool level and user specified priority. If the residual release rate is sent to the outlet works first, the residual release rate above the outlet works capacity for the particular pool level is then sent to the spillway. The spillway branch of the event tree from the Ft. Peck Project, Outlet Works Modification, Major Rehabilitation Evaluation Report was used in this analysis, and is enclosed as Appendix A.

12.7 Economics. The model repeats for a specified number of years of simulation. The specified period of simulation is divided into fifty-year periods. Within each fifty-year period, the damage costs are converted to present value. If a dam failure cost is incurred during any given year, the damage costs the next year and until the end of the fifty-year period are set a pre-described damage cost. If the outlet works are stuck open and result in a loss of pool, the rest of the fifty-year period is also set at a pre-described damage cost. The average present value damage cost is calculated by averaging present value damage costs for all 50-year periods.

12.8 Model Verification. Due to the complexity of the simulation model, verification of the model results were based on the different levels listed below.

a. **Time Series Plotting.** The simulation model automatically plots the generated time series of inflows, pool levels, releases, and damage costs as a quick means of examining the model results. The time series data can be examined for any unexplained trends or shifts in the basic generated data. The time series data can also be examined for interdependence between different time series variables. For example, periods of high releases should correspond to periods of high pool stages and large inflows. Correspondingly, periods of high damage costs should correspond to periods of high releases.

b. **Generated Data Probability Analysis.** The simulation model automatically calculates and plots pool-probability curves and inflow-probability curves for the generated data. The generated data pool-probability curve is plotted along side the pool-probability curve derived from historic data. Likewise, the inflow-probability curves are plotted for both generated and historic data. Plotting the curves insures that the generated data has the same statistical properties as the parent population.

c. **Reservoir Operations.** The simulation model accounts for all release mechanisms and combinations of release mechanisms that are used in the simulation.

Table 5 **Release Summary**	
Ratio of Time Just PP Used	0.81965
Ratio of Time PP and SP Used	0.17760
Ratio of Time Just SP Used	0.00255
Ratio of Time Just Outlet Works Used	0.00010
Ratio of Time PP and Outlet Works Used	0.00000
Ratio of Time Outlet Works and SP Used	0.00000
Ratio of Time PP, OW, and SP Used	0.00000
Ratio of Time Emergency Drawdown	0.00001
TOTAL	1.00000
Where: PP – Power Plant SP – Spillway OW – Outlet Works	

The summary table is useful in verification of the model because it can be compared against historic release records. For the twenty-five years the mainstem system has been operated as a complete system, the maximum annual required release rate has been able to be passed through power plant twenty-two of the twenty-five years or 88% of the years. This compares favorably with the model's 82%. Overall, the summary table allows for quick checks of where the releases are being made.

12.9 Model Sensitivity. Because of the complexity of the model, several sensitivity analyses have been performed on the model. The sensitivity analyses perform two functions; one is to determine if the model gives results which may be rationally supported, and the other is to determine which parameters are most sensitive in affecting the results of the model.

12.9.1 Number of Years of Simulation. The number of years in the model simulation is an important parameter because two counteracting statistical processes are occurring in the model. The number of years determine when an equilibrium or stochastic convergence is attained in the simulation. The first statistical process is the random component of the model may come up with a "hit" or a high damage cost-low probability occurrence after only a few year of simulation. This artificially skews the Benefit-cost ratio on the high side. The countering statistical process is that after a sustained period of simulation, the effect these high damage costs are diminished or diluted by the longer time period. These two countering processes eventually balance out each other and reach a stable solution.

12.9.2 Random Number Generator Seed Number. The random number generator requires a seed number to begin generating a series of uniform random numbers. The sequence of generated random numbers will influence damage costs and the benefit-cost ratio until a certain number of years of simulation until stochastic convergence is reached. The sequence of generated numbers, consequently, is affected by the beginning number or seed number. Sensitivity runs reveal that after 100,000 years of simulation, all sequences of generated values converge to approximately the same number.

12.10 Spillway Damage Probabilities. The probabilities of the particular damage categories occurring during any given year are shown below for a range of spillway flows for the base or existing conditions and the three rehabilitation alternatives.

In general, the damage probabilities reflect that for large flows, there is a corresponding higher probability of sustaining damage to the spillway. The probability of damage decreases with more efficient means of rehabilitation of the spillway.

Table 6 Spillway Damage Probabilities					
		Damage Probability			
Spillway Peak Flow Range (cfs)	**Damage Category**	**Base Condition**	**Rehab Concept #1***	**Rehab Concept #2****	**Rehab Concept #3*****
0-60,000	None	1.0	1.0	1.0	1.0
	Minor	0.0	0.0	0.0	0.0
	Major	0.0	0.0	0.0	0.0
	Tot. Failure	0.0	0.0	0.0	0.0
60,000-90,000	None	0.5	1.0	1.0	1.0
	Minor	0.5	0.0	0.0	0.0
	Major	0.0	0.0	0.0	0.0
	Tot. Failure	0.0	0.0	0.0	0.0
90,000-120,000	None	0.1	1.0	1.0	0.6
	Minor	0.5	0.0	0.0	0.4
	Major	0.4	0.0	0.0	0.0
	Tot. Failure	0.0	0.0	0.0	0.0
120,000-150,000	None	0.0	1.0	1.0	0.5
	Minor	0.4	0.0	0.0	0.3
	Major	0.5	0.0	0.0	0.2
	Tot. Failure	0.1	0.0	0.0	0.0
>150,000	None	0.0	0.8	0.8	0.4
	Minor	0.0	0.1	0.1	0.3
	Major	0.1	0.1	0.1	0.2
	Tot. Failure	0.9	0.0	0.0	0.1
* -Stilling Basin, Slab Anchors ** -Flip Bucket, Slab Anchors, Cutoff Walls + Wingwalls *** -Flip Bucket, Seal Joints, Cutoff Walls + Wingwalls					

12.11 Results of Monte-Carlo Simulation. The results of the Monte-Carlo simulations are shown below for an array of total failure costs and for the base conditions and the three design alternatives: stilling basin and anchors, flip bucket and anchors, and flip bucket and joint sealing.

Table 7
Monte Carlo Simulation Results

Total Failure Damage Cost	Alternative	Average Present-Value Cost ($)	Rehab Cost ($)	Net Benefit ($)	B/C Ratio
101,073,000[1]	Base Cond.	22,019,000	na	na	na
94,487,000[2]	Base Cond.	21,745,000	na	na	na
34,449,000[3]	Base Cond.	19,250,000	na	na	na
101,073,000[1]	Stilling Basin	455,000	75,642,000	21,564,000	0.29
94,487,000[2]	and	455,000	75,642,000	21,290,000	0.28
34,449,000[3]	Anchors	455,000	75,642,000	18,795,000	0.25
101,073,000[1]	Flip Bucket	544,000	55,054,000	21,475,000	0.39
94,487,000[2]	and	544,000	55,054,000	21,201,000	0.39
34,449,000[3]	Anchors	544,000	55,054,000	18,706,000	0.34
101,073,000[1]	Flip Bucket	4,039,000	17,572,000	17,980,000	1.02
94,487,000[2]	and	4.007,000	17,572,000	17,738,000	1.01
34,449,000[3]	Sealing	3,719,000	17,572,000	15,531,000	0.88

[1] Total Failure Scenario #1.
[2] Total Failure Scenario #2.
[3] Total Failure Scenario #3.

The first two alternatives have the same average present value damage cost for all total failure damages. This is because the spillway rehab is extensive enough, that there is not any total failure, consequently the damages are constant.

The third alternative, the flip bucket and joint sealing, have higher damage costs, but are more than offset by the considerably lower rehab costs. The benefit to cost ratio for two of the total failure costs reflect that rehabbing the spillway would be economically feasible.

13. Recommended Release Strategy. In order to minimize project risk, a release strategy for flows in excess of power tunnel capacity was developed. The release strategy was developed based on operational issues with both the spillway and outlet works. Assuming the power tunnels release 15,000 cfs, additional flow release of 5,000 to 15,000 cfs for 2-3 months may be required. Employment of a risk minimizing release strategy is recommended until structural modifications are made. Development of release strategy is based on the following:

a. The outlet works have not been operated since 1975. In 1975, considerable inspection and maintenance was performed to insure the integrity of the ring gate during operation.

b. The spillway was successfully operated at a maximum flow rate of 20,000 cfs in the 1970's.

c. After the Engineering Reconnaissance Study was completed by R.W. Beck in August 1997, the spillway was operated in November 1997 with a flow rate from 3,000 to 7,500 cfs for 4 months and the peak flow of 7,500 for 3 weeks in that time span.

d. Spillway failure will be confined to the lower end of the spillway. The worst case scenario destroys most of the spillway but does not result in uncontrolled flow release or threatening the integrity of the dam. The spillway gates and upper section of the spillway should be protected by the low flow velocities and a cutoff wall.

e. Outlet works failure may be catastrophic. The worst case scenario destroys the ring gate(s) and damages the downstream tunnel. Closure of the emergency gate under flow is unsuccessful. Flow release is uncontrolled through the outlet works. Tunnel damage caused by ring gate failure is severe and flowing water is not confined to the lined tunnel. The earthen embankment is eroded and the safety of the dam is threatened.

Above the power tunnel capacity, a list was developed which specifies the release mechanism versus increasing for a flow range. Revision of the flow levels within each bracket may be necessary as operating experience dictates. The following release strategy is recommended.

0-10,000 cfs: Utilize the spillway.

10,000-23,000 cfs: Utilize the spillway. Verify that the drains are functioning and monitor during operation.

20,000-60,000 cfs: Utilize the spillway. Verify drains are functioning, monitor during operation, and perform detailed inspections of the spillway and scour hole after operation.

Above 60,000 cfs: Incorporate the use of flood tunnels when damage to the spillway appears eminent. Operate tunnel number 3 first to maximum capacity before tunnel 4. If objectionable flood tunnel operation results switch back to the spillway and accept the damage which occurs. Perform inspections of the spillway and flood tunnels as necessary after operation.

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